Chapter 7 STORAGE FACILITIES

7.1 INTRODUCTION

The traditional design of storm drainage systems has been to collect and convey storm runoff as rapidly as possible to a suitable location where it can be discharged. As areas urbanize, this type of design may result in major drainage and flooding problems downstream. Under favorable conditions, the temporary storage of some of the storm runoff can decrease downstream flows and often the cost of the downstream conveyance system. Detention storage facilities can range from small facilities contained in parking lots or other on-site facilities to large lakes and reservoirs. This chapter provides general design criteria for detention storage basins as well as procedures for performing preliminary and final sizing and reservoir routing calculations. Recharge basins are a special type of detention basin designed to drain into the groundwater table; these are not addressed in this manual.

The use of storage facilities for stormwater management has increased dramatically in recent years. The benefits of storage facilities can be divided into two major control categories of quality and quantity.

Quality

Control of stormwater quality using storage facilities offers the following potential benefits:

- decrease downstream channel erosion,
- control of sediment deposition,
- improved water quality through stormwater filtration.

Quantity

Controlling the quantity of stormwater using storage facilities can provide the following potential benefits:

- prevention or reduction of peak runoff rate increases caused by urban development,
- mitigation of downstream drainage capacity problems,
- recharge of groundwater resources,
- reduction or elimination of the need for downstream outfall improvements,
- maintenance of historic low flow rates by controlled discharge from storage.

7.1.1 Detention and Retention Facilities Definition

Urban stormwater storage facilities are generally referred to as detention (dry pond) or retention (wet pond) facilities. Dry ponds are those that are designed to reduce the peak discharge and only detain runoff for some short period of time. They are designed to completely drain after the design storm has passed. Wet ponds are designed to contain a permanent pool of water usually for water quality purposes.

7.1.2 Concept Definitions

Following are discussions of concepts which are important in storage design:

Outlet Structures	Outlet structures selected for storage facilities typically include a principal outlet and an emergency overflow, and must be able to accomplish the design functions of the facility. Outlet structures can take the form of combinations of drop inlets, pipes, weirs, and orifices. The principal outlet is intended to convey the design storm.
Stage-storage Curve	A stage-storage curve defines the relationship between the depth of water and storage volume in a reservoir.
Stage-discharge Curve	A stage-discharge curve defines the relationship between the depth of water and the discharge or outflow from a storage facility. A typical storage facility has two outlets: principal outlet and emergency overflow.
Inflow, I	Discharge into a storage facility at a specific time.
Outflow, O	Discharge out of a storage facility at a specific time.
Hydrograph	The hydrograph is a graph of the time distribution of flow rate at a single point.

7.2 DESIGN CRITERIA

Storage may be concentrated in large basin-wide or regional facilities or distributed throughout an urban drainage system. Possible dispersed or on-site storage may be developed in depressed areas in parking lots, road embankments and freeway interchanges, parks and other recreation areas, and small lakes, ponds and depressions within urban developments. Although there are no known strict rules covering ponding adjacent to roadways, ponding within the clear zone and adjacent R/W should be evaluated for safety on a project by project basis. The utility of any storage facility depends on the amount of storage, its location within the system, and its operational characteristics. An analysis of such storage facilities should consist of comparing the design flow at a point or points downstream of the proposed storage site with and without storage. In addition to the design flow, other flows in excess of the design flow that might be expected to pass through the storage facility should be included in the analysis.

The design criteria for storage facilities should include:

- location,
- outflow rate,
- storage volume,
- grading and depth requirements,
- outlet structure.

7.2.1 Location

It should be noted that the location of storage facilities is very important as it relates to the effectiveness of these facilities to control downstream flooding. Small facilities will only have minimal flood control benefits and these benefits will quickly diminish as the flood wave travels downstream. Multiple storage facilities located in the same drainage basin will affect the timing of the runoff through the conveyance system which could decrease or increase flood peaks in different downstream locations. Thus it is important for the designer to design storage facilities as a drainage structure that both controls runoff from a defined area and interacts with other drainage structures within the drainage basin. Effective stormwater management must be coordinated on a regional or basin-wide planning basis. Local stormwater management should be coordinated with the local entities such as watershed districts, municipalities and counties, judicial and county ditch authorities.

7.2.2 Storage

Storage volume shall normally be provided to attenuate the post-development peak discharge rates to pre-developed peak discharge rates for the design storm, or other design storms depending on what the downstream system is designed for. Routing calculations must be used to demonstrate that the storage volume is adequate. If sedimentation during construction causes loss of detention volume, design dimensions shall be restored before completion of the project.

Dry Ponds

Careful finish grading is required to avoid creation of upland surface depressions that may retain runoff. The bottom area of storage facilities should be graded toward the outlet to prevent standing water conditions. A minimum 2 percent bottom slope is recommended. A low flow or pilot channel constructed across the facility bottom from the inlet to the outlet is recommended to convey low flows, and prevent standing water conditions.

Wet Ponds

The maximum depth of permanent storage facilities will be determined by site conditions, design constraints, and environmental needs. In general, if the facility provides a permanent pool of water, a depth sufficient to discourage growth of weeds (without creating undue potential for anaerobic bottom conditions) should be considered. A depth of 4 feet is generally reasonable. Where aquatic habitat is required, wildlife experts should be contacted for site specific criteria relating to such things as depth, habitat, and bottom and shore geometry.

7.2.3 Flow Rate

For large storage facilities, selecting a flood magnitude for sizing the emergency outlet should be consistent with the potential threat to downstream life and property if the basin embankment were to fail. The sizing of a particular outlet structure shall be based on results of hydrologic routing calculations. Control structure outflow rates shall usually approximate pre-developed peak runoff rates for the design storm or what is required by the local regulatory agency. It is recommended that emergency overflow be provided for events greater than the design storm. In addition to controlling the peak discharge from the outlet structures, storage facilities will change the timing of the entire hydrograph. If several storage facilities are located within a particular basin it is important to determine what effects a particular facility may have on combined hydrographs in downstream locations. The location and timing of the structure should be coordinated with the local drainage offices.

7.3 STORAGE FACILITY ANALYSIS

The following data will be needed to complete storage design and routing calculations:

- Inflow hydrograph for selected design storms.
- Stage-storage curve for proposed storage facility (Figure 7.1).
- Stage-discharge curve for all outlet control structures (Figure 7.2).

A routing procedure is used to route the inflow hydrograph through the storage facility with different basin (stage-storage curve) and outlet geometry (stage-discharge curve) until the desired outflow hydrograph is achieved.

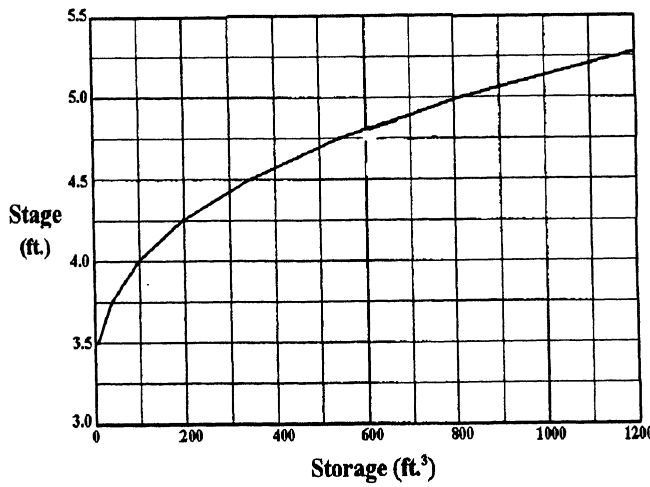


Figure 7.1 Example Stage-Storage Curve

7.3.1 Stage-Storage Curve

A stage-storage curve defines the relationship between the depth of water and storage volume in a reservoir. The data for this type of curve are usually developed using a topographic map and the double-end area, frustum of a pyramid or prismoidal formulas. Only the more commonly used double-end area formula is provided. To convert from ft^3 to acre-ft, divide number by 43560.

The double-end area formula is expressed as:

$$V_{1,2} = \left[\frac{(A_1 + A_2)}{2}\right] d \tag{7.1}$$

Where:
- $V_{1,2}$ = storage volume between elevations 1 and 2 (ft^3)
- A_1 = surface area at elevation 1 (ft^2)
- A_2 = surface area at elevation 2 (ft^2)
- d = change in elevation between points 1 and 2 (ft)

7.3.2 Stage-Discharge Curve

A stage-discharge curve defines the relationship between the depth of water and the discharge or outflow from a storage facility. A typical storage facility has two outlets: principal outlet and emergency overflow. The principal outlet is usually designed with a capacity sufficient to convey the design flood without allowing flow to enter the emergency spillway. A pipe culvert, weir, or other appropriate outlet can be used for the principal outlet. The emergency overflow is sized to provide a bypass for floodwater during a flood that exceeds the design capacity of the principal outlet. The stage-discharge curve should take into account the discharge characteristics of both the principal outlet and emergency overflow. Care should be taken to consider downstream tailwater effects, especially in cases of multiple ponds where backwater may occur.

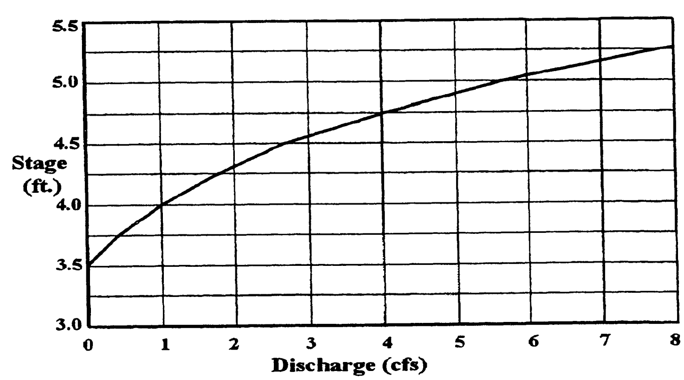


Figure 7.2 Example Stage-Discharge Curve

7.3.3 Generalized Routing Procedure

A general procedure for using the above data in the design of storage facilities is presented below. This procedure can involve a significant number of reservoir routing calculations to obtain the desired results.

Step 1 Compute inflow hydrograph for runoff from the design storm using the procedures outlined in the Hydrology Chapter. Determine allowable peak outflow rate.

Step 2 If necessary perform preliminary calculations to evaluate detention storage requirements for the hydrograph from Step 1. If storage requirements are satisfied for runoff from the design storms, runoff from lower return period storms is assumed to be controlled.

Step 3 Determine the physical dimensions necessary to hold the estimated volume from Step 2, including freeboard. The maximum storage requirement calculated from Step 2 should be used.

Step 4 Size the outlet structure. The estimated peak stage will occur for the estimated volume from Step 2. The outlet structure should be sized to convey the allowable discharge at this stage.

Step 5 Perform routing calculations using inflow hydrographs from Step 1 to check the preliminary design using the storage routing equations. If the routed peak discharges from the design storm exceeds the allowable peak discharges, or if the peak stage varies significantly from the estimated peak stage from Step 4, then revise the estimated volume and return to step 3.

Step 6 Consider emergency overflow from runoff due to a larger storm than the design storm and established freeboard requirements.

Step 7 Evaluate the downstream effects of detention outflow to ensure that the routed hydrograph does not cause downstream flooding problems.

Step 8 Evaluate the control structure outlet velocity and provide channel and bank stabilization if the velocity will cause erosion problems downstream.

7.4 OUTLET HYDRAULICS

Equations are provided for sharp-crested weirs, broad-crested weirs, v-notch weirs, proportional weirs, and orifices, or combinations of these facilities. If culverts are used as outlet works, procedures presented in the Culvert Chapter should be used to develop stage-discharge data. When analyzing release rates the tailwater influence of the principal spillway culvert on the control structure (orifice and/or weirs) must be considered to determine the effective head on each opening.

7.4.1 Sharp Crested Weirs

Sharp-crested weir flow equations for no end contractions, two end contractions, and submerged discharge conditions are presented below.

Sharp Crested Weir with No End Contractions

A sharp-crested weir with no end contractions is illustrated in Figure 7.3. The discharge equation for this configuration is (Chow, 1959):

$$Q = \left[3.27 + 0.4\left(\frac{H}{H_c}\right)\right] L H^{1.5} \tag{7.4}$$

Where: Q = discharge (cfs)
H = head above weir crest excluding velocity head (ft)
H_c = height of weir crest above channel bottom (ft)
L = horizontal weir length (ft)

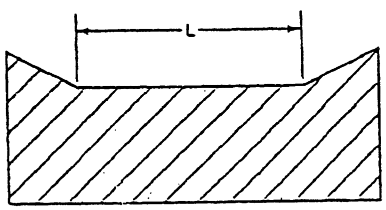


Figure 7.3 Sharp-Crested Weir (No End Contractions)

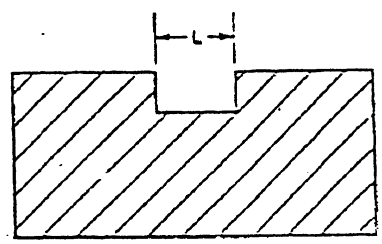


Figure 7.4 Sharp-Crested Weir, (Two End Contractions)

Sharp Crested Weir with Two End Contractions

A sharp-crested weir with two end contractions is illustrated in Figure 7.4. The discharge equation for this configuration is (Chow, 1959):

$$Q = \left[3.27 + 0.4\left(\frac{H}{Hc}\right)\right](L - 0.2H)\, H^{1.5} \tag{7.5}$$

Where: Q = discharge (cfs)
H = head above weir crest excluding velocity head (ft)
H_c = height of weir crest above channel bottom (ft)
L = horizontal weir length (ft)

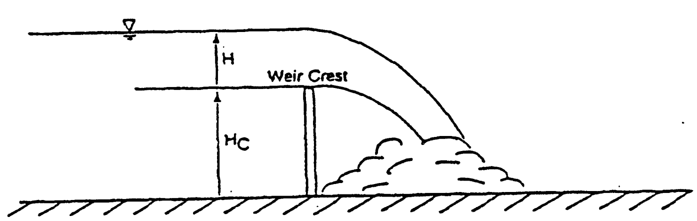


Figure 7.5 Sharp-Crested Weir and Head

Submerged Sharp Crested Weir

A sharp-crested weir will be affected by submergence when the tailwater rises above the weir crest elevation. The result will be that the discharge over the weir will be reduced. The discharge equation for a sharp-crested submerged weir is (Brater and King, 1976):

$$Q_s = Q_f\left[1 - \left(\frac{H_2}{H_1}\right)^{1.5}\right]^{0.385} \tag{7.6}$$

Where: Q_s = submergence flow (cfs)
Q_f = free flow (cfs)
H_1 = upstream head above crest (ft)
H_2 = downstream head above crest (ft)

7.4.2 Broad Crested Weirs

The equation generally used for the broad-crested weir is (Brater and King, 1976):

$$Q = CLH^{1.5} \tag{7.7}$$

Where: Q = discharge (cfs)
C = broad-crested weir coefficient
L = broad-crested weir length (ft)
H = head above weir crest, ft

If the upstream edge of a broad-crested weir is so rounded as to prevent contraction and if the slope of the crest is as great as the loss of head due to friction, flow will pass through critical depth at the weir crest; this gives the maximum C value of 3.087. For sharp corners on the broad-crested weir, a minimum C value of 2.6 should be used. Additional information on C values as a function of weir crest breadth and head is given in Table 7.1.

Table 7.1 Broad-Crested Weir Coefficient C Values as a Function of Weir Crest Breadth and Head (ft)

Measured Head, H[1] (ft)	Breadth Of The Crest Of Weir (ft)										
	0.50	0.75	1.00	1.50	2.00	2.50	3.00	4.00	5.00	10.00	15.00
0.2	2.80	2.75	2.69	2.62	2.54	2.48	2.44	2.38	2.34	2.49	2.68
0.4	2.92	2.80	2.72	2.64	2.61	2.60	2.58	2.54	2.50	2.56	2.70
0.6	3.08	2.89	2.75	2.64	2.61	2.60	2.68	2.69	2.70	2.70	2.70
0.8	3.30	3.04	2.85	2.68	2.60	2.60	2.67	2.68	2.68	2.69	2.64
1.0	3.32	3.14	2.98	2.75	2.66	2.64	2.65	2.67	2.68	2.68	2.63
1.2	3.32	3.20	3.08	2.86	2.70	2.65	2.64	2.67	2.66	2.69	2.64
1.4	3.32	3.26	3.20	2.92	2.77	2.68	2.64	2.65	2.65	2.67	2.64
1.6	3.32	3.29	3.28	3.07	2.89	2.75	2.68	2.66	2.65	2.64	2.63
1.8	3.32	3.32	3.31	3.07	2.88	2.74	2.68	2.66	2.65	2.64	2.63
2.0	3.32	3.31	3.30	3.03	2.85	2.76	2.72	2.68	2.65	2.64	2.63
2.5	3.32	3.32	3.31	3.28	3.07	2.89	2.81	2.72	2.67	2.64	2.63
3.0	3.32	3.32	3.32	3.32	3.20	3.05	2.92	2.73	2.66	2.64	2.63
3.5	3.32	3.32	3.32	3.32	3.32	3.19	2.97	2.76	2.68	2.64	2.63
4.0	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.70	2.64	2.63
4.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.74	2.64	2.63
5.0	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.64	2.63
5.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.64	2.63

[1]Measured at least 2.5H upstream of the weir.

Source: Brater and King (1976).

7.4.3 V-notch Weirs

The discharge through a v-notch weir can be calculated from the following equation (Brater and King, 1976).

$$Q = 2.5 \tan\left(\frac{\theta}{2}\right) H^{2.5} \tag{7.8}$$

Where: Q = discharge (cfs)
θ = angle of v-notch (degrees)
H = head on apex of notch (ft)

7.4.4 Orifices

An orifice is an opening through which water may flow. Generally the walls are assumed to be thin with a square edge, though variations in geometry are possible. The entrance coefficient will vary with geometry, but is typically given a value of 0.6 for circular orifices with square edged entrances. If the orifice discharges as a free outfall then the head, H is measured from the upstream water surface to the center of the orifice. If the orifice is submerged the head on the pipe is the difference between the upstream and downstream water surfaces. Pipes smaller than 12" in diameter may be analyzed as a submerged orifice if H/D is greater than 1.5. Pipes with a diameter larger than 12" should be analyzed as a discharge pipe with headwater and tailwater effects taken into account, not just as an orifice.

$$Q = CA\left(2gH\right)^{0.5} \tag{7.9}$$

Where: Q = discharge (cfs)
C = entrance coefficient (typically C = 0.6)
A = cross-section area of pipe (ft^2)
g = acceleration due to gravity (32.2 ft/s^2)
D = diameter of pipe (ft)
H = head on pipe (ft)

7.5 DESIGN PROCEDURE

Routing calculations needed to design storage facilities, although not extremely complex, are time consuming and repetitive. To assist with these calculations there are many available reservoir routing computer programs. Also, the storage indicator method can be used which makes calculations simple. All storage facilities shall be designed and analyzed using reservoir routing calculations. Software available for final routing includes TR20, HEC-1 and HEC-RAS. Many commercial programs are also available but the engineer/technician operating the programs should possess the background and skill necessary to correctly interpret the results.

7.5.1 Preliminary Detention Calculation of Storage Volume

A preliminary estimate of the storage volume required for peak flow attenuation may be obtained from a simplified design procedure that replaces the actual inflow and outflow hydrographs with the standard triangular shapes shown in Figure 7.6. This procedure is not recommended for final design but only where preliminary estimates are needed of storage volume.

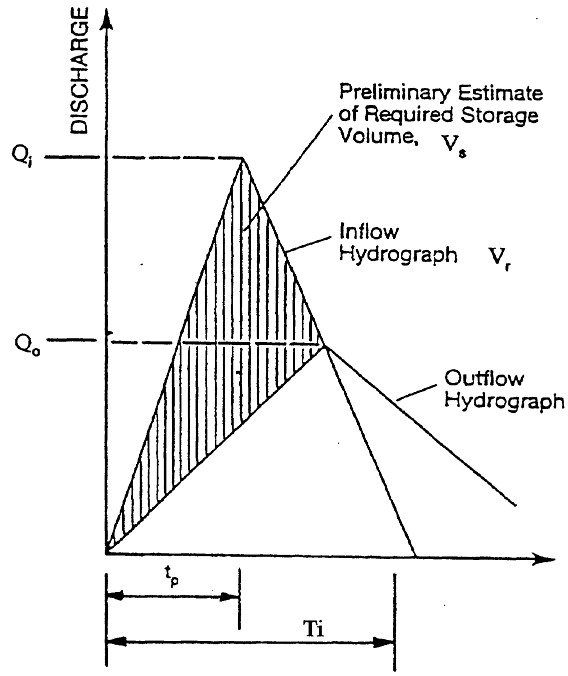


Figure 7.6 Triangular Shaped Hydrographs
(For Preliminary Estimate Of Required Storage Volume)

Use inflow hydrograph to determine Q_i and T_i. The allowable peak outflow rate, Q_o must be estimated. The peak discharge for pre-existing conditions is commonly used.

The required storage volume may be estimated from the area above the outflow hydrograph and inside the inflow hydrograph, expressed as:

$$V_S = 0.5\, Ti \left(Q_i - Q_O\right) \tag{7.10}$$

Where: V_S = storage volume estimate (ft^3)
Q_i = peak inflow rate (cfs)
Q_o = peak outflow rate (cfs)
T_i = duration of basin inflow (sec)

7.5.2 Storage Indicator Method

The most commonly used method for routing inflow hydrograph through a detention pond is the Storage Indication or modified Puls method. This method begins with the continuity equation which states that the inflow minus the outflow equals the change in storage (I - O = ΔS). By taking the average of two closely spaced inflows and two closely spaced outflows, the method is expressed by Equation 7.11. In Equation 7.11 subscript 1 refers to the beginning and subscript 2 refers to the end of the time interval.

$$\frac{\Delta S}{\Delta t} = \frac{(I_1+I_2)}{2} - \frac{(O_1+O_2)}{2} \tag{7.11}$$

Where: ΔS = change in storage (ft^3)
Δt = time interval (minutes)
I = inflow (ft^3)
O = outflow (ft^3)

Equation 7.11 can be rearranged so that all the known values are on the left side of the equation and all the unknown values are located on the right hand side of the equation, as shown in Equation 7.12. Now, the equation with two unknowns, S_2 and O_2, can be solved with one equation. The following procedure can be used to perform routing through a reservoir or storage facility using Equation 7.12.

$$\frac{(I_1+I_2)}{2} + \left(\frac{S_1}{\Delta t}+\frac{O_1}{2}\right) - O_1 = \left(\frac{S_2}{\Delta t}+\frac{O_2}{2}\right) \qquad (7.12)$$

Where:
- I_1 = Instantaneous inflow to the pond at the beginning of the incremental time period, Δt
- I_2 = Instantaneous inflow at the end of the time period, Δt
- O_1 = Instantaneous outflow at the beginning of the time period, Δt
- O_2 = Instantaneous outflow at the end of the time period, Δt
- S_1 = Storage volume in the pond at the beginning of the incremental time period, Δt.
- S_2 = Storage volume in the pond at the end of the incremental time period, Δt
- Δt = Incremental routing time interval selected to subdivide hydrograph into finite time elements

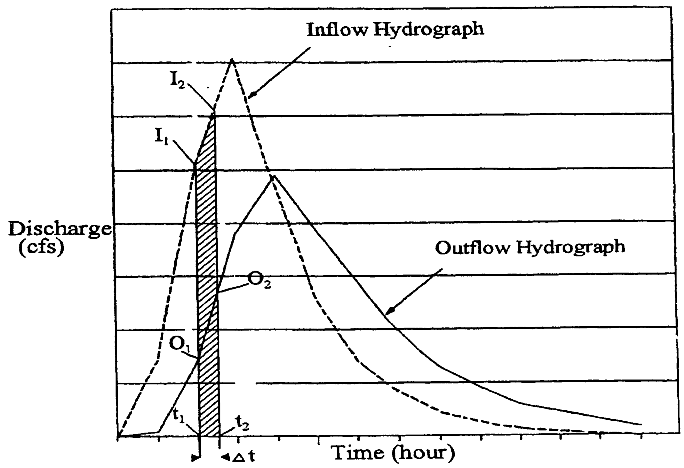


Figure 7.7 Routing Hydrograph Schematic

Step 1 Develop an inflow hydrograph, stage-discharge curve, and stage-storage curve for the proposed storage facility.

Step 2 Select a routing time period, Δt, to provide a minimum of five points on the rising limb of the inflow hydrograph.

Step 3 Use the stage-storage and stage-discharge data from Step 1 to develop a storage indicator numbers table that provides storage indicator values, $(S/\Delta t) + O/2$, versus stage. A typical storage indicator numbers table contains the following column headings:

STORAGE INDICATOR TABLE

(1) Stage (ft)	(2) Discharge O (cfs)	(3) Storage S (ft³)	(4) $O_2/2$ (cfs)	(5) $S_2/\Delta t$ (cfs)	(6) $S_2/\Delta t + O_2/2$ (cfs)

A. The discharge (O) and storage (S) are obtained from the stage-discharge and stage-storage curves, respectively.

B. The subscript 2 is arbitrarily assigned at this time.

C. The time interval (Δt) must be the same as the time interval used in the tabulated inflow hydrograph.

Step 4 Develop a storage indicator numbers curve by plotting the outflow (column 2) vertically against the storage indicator numbers in column (6). An equal value line plotted as $O_2 = S_2/\Delta t + O_2/2$ should also be plotted. If the storage indicator curve crosses the equal value line, a smaller time increment (Δt) is needed.

Step 5 A supplementary curve of storage (column 3) vs. $S_2/\Delta t + O_2/2$ (column 4) can also be constructed. This curve does not enter into the mainstream of the routing; however, it is useful for identifying storage for any given value of $S_2/\Delta t + O_2/2$. A plot of storage vs. time can be developed from this curve.

Step 6 The routing can now be performed by developing a routing table for the solution of Equation 7.12. A typical storage indicator numbers table contains the following column headings:

ROUTING TABLE

(1) Time (Hour)	(2) Inflow (cfs)	(3) $(I_2 + I_1)/2$ (cfs)	(4) $S_1/\Delta t + O_1/2$ (cfs)	(5) O_1 (cfs)	(6) $S_2/\Delta t + O_2/2$ (cfs)	(7) O_2 (cfs)

A. Columns (1) and (2) are obtained from the inflow hydrograph.

B. Column (3) is the average inflow over the time interval.

C. The initial values for columns (4) and (5) are generally assumed to be zero since there is no storage or discharge at the beginning of the hydrograph when there is no inflow into the basin.

D. The left side of the Equation 7.12 is determined algebraically as columns (3) + (4) - (5). This value equals the right side of equation 7.12 or $S_2/\Delta t + O_2/2$. And is placed in column (6).

E. Enter the storage indicator curve with $S_2/\Delta t + O_2/2$ (column 6) to obtain O_2 (column 7).

F. Column (6) ($S_2/\Delta t + O_2/2$) and column (7) (O_2) are transported to the next line and become $S_1/\Delta t + O_1/2$ and O_1 in columns (4) and (5), respectively. Because ($S_2/\Delta t + O_2/2$) and O_2 are the ending values for the first time step, they can also be said to be the beginning values for the second time step.

G. Columns (3), (4), and (5) are again combined and the process is continued until the storm is routed.

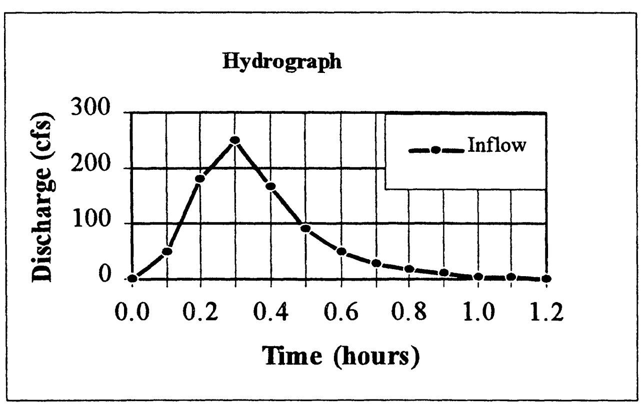
H. Peak storage depth and discharge (O_2 in column (7)) will occur when column (6) reaches a maximum. The storage indicator numbers table developed in Step 3 is entered with the maximum value of $S_2/\Delta t + O_2/2$ to obtain the maximum amount of storage required. This table can also be used to determine the corresponding elevation of the depth of stored water.

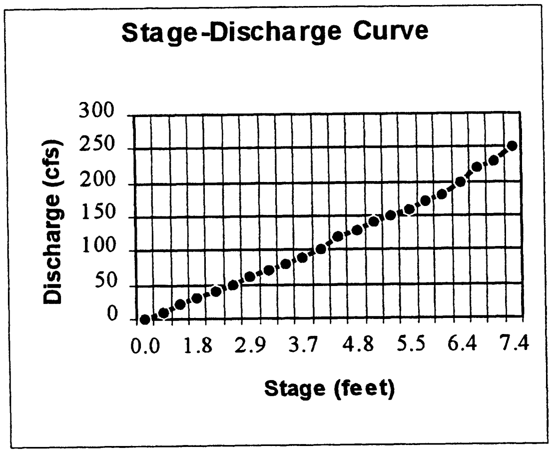
I. The designer needs to make sure the peak value in column (7) does not exceed the allowable discharge as prescribed by the stormwater management criteria.

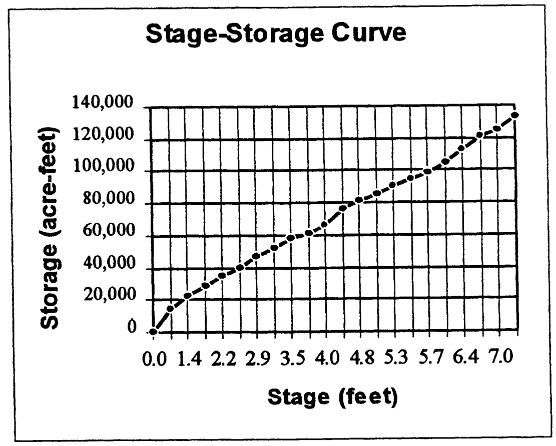
Step 7 Plot O_2 (column(7)) versus time (column (1)) to obtain the outflow hydrograph.

7.5.3 Routing Example

This example demonstrates the application of the methodology presented in this chapter for the design of a typical detention storage facility. An inflow hydrograph with a peak discharge of 250 cfs is provided. The inflow hydrograph will typically be developed using hydrologic methods from the Hydrology Chapter. Stage-discharge and stage-storage curves are developed for the storage facility.







Once the stage-discharge and stage-storage curves are determined for the storage facility a storage indicator table is developed. For a each stage (column 1) a corresponding discharge (column 2) is taken from the stage-discharge curve and storage (column 3) is taken from the stage-storage curve. The remainder of the storage indicator table is filled out according to the procedure given in Section 7.5.2, Step 3.

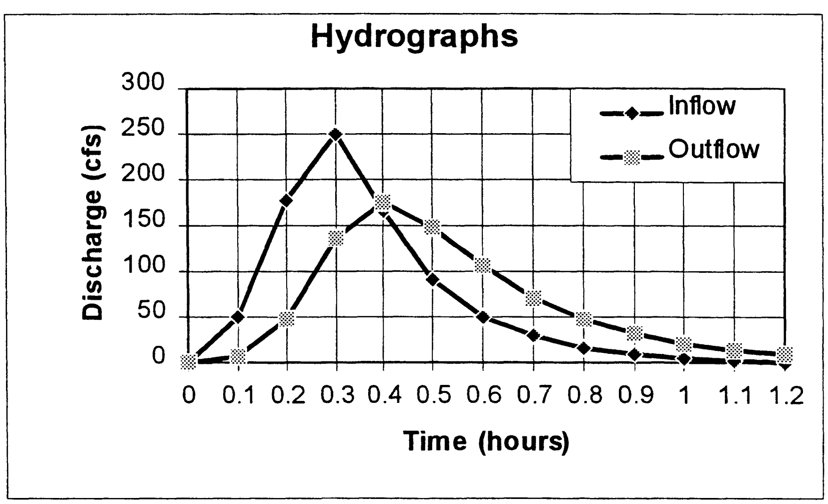
STORAGE INDICATOR TABLE

(1) Stage (ft)	(2) Discharge, O (cfs)	(3) Storage, S (ft^3)	(4) $O_2/2$ (cfs)	(5) $S_2/\Delta t$ (cfs)	(6) $S_2/\Delta t + O_2/2$ (cfs)
0.0	0	0	0	0	0
0.9	10	13872.45	5	38.53	43.53
1.4	20	21842.07	10	60.67	70.67
1.8	30	28354.82	15	78.76	93.76
2.2	40	34990.65	20	97.20	117.20
2.5	50	40048.98	25	111.25	136.25
2.9	60	46902.95	30	130.29	160.29
3.2	70	52126.21	35	144.80	179.80
3.5	80	57420.97	40	159.50	199.50
3.7	90	60990.76	45	169.42	214.42
4.0	100	66405.76	50	184.46	234.46
4.5	120	75592.76	60	209.98	269.98
4.8	130	81202.94	65	225.56	290.56
5.0	140	84984.20	70	236.07	306.07
5.3	150	90718.14	75	251.99	326.99
5.5	160	94582.35	80	262.73	342.73
5.7	170	98479.98	85	273.56	358.56
6.0	180	104389.4	90	289.97	379.97
6.4	200	112387.1	100	312.19	412.19
6.8	220	120521.0	110	334.78	444.78
7.0	230	124639.5	115	346.22	461.22
7.4	250	132980.1	125	369.39	494.39

After the storage indicator table is completed, a routing table is developed. The inflow hydrograph is entered time (column 1) and inflow (column 2). The remainder of the table is filled out according to the procedure given in Section 7.5.2, Step 6. The outflow (column 7) is then analyzed to make sure the design criteria were met.

ROUTING TABLE						
(1) Time (Hour)	(2) Inflow (cfs)	(3) $(I_2 + I_2)/2$ (cfs)	(4) $S_1/\Delta t + O_1/2$ (cfs)	(5) O_1 (cfs)	(6) $S_2/\Delta t + O_2/2$ (cfs)	(7) O_2 (cfs)
0	0	0	0	0	0	0
0.1	50	25	0	0	25	5.74
0.2	178	114	25	5.74	133.26	48.43
0.3	250	214	133.26	48.43	298.83	135.33
0.4	165	207.5	298.83	135.33	371	175.81
0.5	90	127.5	371.00	175.81	322.69	147.94
0.6	50	70	322.69	147.94	244.75	105.79
0.7	29	39.5	244.75	105.79	178.46	69.32
0.8	16	22.5	178.46	69.32	131.64	47.58
0.9	9	12.5	131.64	47.58	96.56	31.19
1.0	5	7	96.56	31.19	72.37	20.73
1.1	3	4	72.37	20.73	55.64	14.46
1.2	1	2	55.64	14.46	43.18	9.92

Typically a storage facility will be analyzed for a series of storm events and drainage conditions. The designer is interested in pre-development and post-development hydrographs. A common design criteria is that the pre-development peak discharge will not exceed the post-development peak discharge after being routed through the storage facility. The designer may evaluate multiple events, the design event (10-50 year) is analyzed and a major flood event (typically 100 year) is checked to insure the storage facility does not cause flood damage. The results from the calculations are compared against the design criteria, if the storage facility fails to meet the design criteria, it must be re-designed by changing the geometry and/or outlet design. This is an iterative procedure, and continues until all design goals are met.



7.6 REFERENCES

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